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Design of intermediate transverse stiffeners for shear web panels

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ABSTRACT

The function of intermediate transverse stiffeners attached on plate girder web panels is to provide simple supports so as to reduce the panel aspect ratio, thereby increasing the shear buckling strength. This study investigates the moments of inertia of the transverse stiffeners required to maintain the simple support condition not just for elastic buckling but also for postbuckling through linear buckling and nonlinear finite element analyses. From nonlinear regression analyses of the FEA results, a set of design equations are formulated and compared with the current provisions in AISC and AASHTO LRFD specifications. It is found that the design equation for elastic buckling in AISC and AASHTO LRFD specifications is too conservative especially when the aspect ratio is greater than 1.0. Also, the design equation for postbuckling in AISC and AASHTO specifications results in designs that are too conservative due to oversimplification.

1. Introduction

Approaches toward the design of intermediate transverse stiffeners are directly related to the design shear strength of the web panel. It is a common practice to utilize postbuckling strength for an economical design of civil engineering structures such as bridges by using relatively thin web panels. The transverse stiffeners are to be designed so as to ensure that the web panel is able to develop its design shear strength including the postbuckling strength. The methodology to assess the design shear strength differs from code to code and so does the design of the stiffeners.

Before postbuckling strength was reflected in the design of plate girders, the stiffeners were designed to have a proper moment of inertia required to provide nodal lines (simply supported boundaries) during elastic buckling. Timoshenko [1] obtained the flexural rigidity of the transverse stiffener that ensures a straight boundary for the shear web panel during elastic buckling. Later, Stein and Fralich [2] reported that the numerical results obtained by Timoshenko [1] may be in substantial error, because the assumed displacement expressed in an infinite series of harmonic functions is not suitable to express the deformation of the stiffened plate when using a limited number of terms. Stein and Fralich [2] obtained a better solution for simply supported, infinitely long plates reinforced by equally spaced transverse stiffeners. Bleich [3] developed an approximate formula based on the results obtained in [2], which is adopted in the current AISC and AASHTO specifications [4,5]. Despite the fact that Stein and Fralich [2] investigated limited aspect ratios of $d_o/D = 0.2$, 0.5, and 1.0, Bleich's formula [3] has been applied to aspect ratios greater than 1.0 without theoretical proof.

AISC [6] for the first time appreciated postbuckling strength in the design of plate girders following Basler [7], and specified the additional cross-sectional area requirement to resist the compressive axial force due to tension field action. Thereafter AASHO [8] followed suit. However, numerous studies (Rahal and Harding [9]; Stanway et al. [10,11]; Xie and Chapman [12]; Lee et al. [13]; Kim et al. [14]) reported that large compressive axial forces did not develop in the stiffeners during postbuckling. Yoo and Lee [15] and Lee et al. [16,17] shed light on the reason why the stiffeners are not necessarily subjected to direct compression in normal plate girders. The area requirement was finally removed from AASHTO and AISC Specifications in the 2007 and 2010 editions [18,19], respectively, while Eurocode 3 [20] and BS5400 [21] still consider the axial force.

Transverse stiffeners undergo bending due to severe out-of-plane deformations of the web panel during postbuckling [13]. Massonnet [22] reported that for the stiffeners on his test girders to remain straight and fully effective up to the collapse of the girders, it was necessary that the flexural rigidity of the stiffeners be increased by several times that required to maintain the nodal line during elastic buckling. Ŝkaloud [23] reported: "If it is required that the stiffener should remain practically straight in the whole postbuckling range, it is necessary to increase its flexural rigidity 3

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Nomenclature

b	the smaller of d_o and D
C	ratio of the shear buckling resistance to the shear yield strength
D	depth of the girder
d_o	transverse stiffener spacing
E	modulus of elasticity
F_{crs}	local buckling stress of the stiffener
F_{ys}	specified minimum yield strength of the stiffener
F_{yw}	specified minimum yield strength of the web
I_t	moment of inertia of the transverse stiffener
j	coefficient for moment of inertia of transverse stiffener for elastic buckling
j_y	coefficient for moment of inertia of transverse stiffener for yield zone
k	shear buckling coefficient
n	multiplication factor for moment of inertia of transverse stiffener in the postbuckling range
R_y	reduction factor for moment of inertia in yield zone
t_w	thickness of the web
V_p	shear yield strength
μ	Poisson's ratio
ρ_t	the larger of F_{yw} or F_{crs}

times." Lee et al. [24] suggested a simple design rule from a parametric study based on nonlinear finite element analyses and experimental results: "The moment of inertia of the transverse stiffener needs to be increased 6 times for the web panel to develop at least 97.5% of its potential postbuckling strength." The design rule suggested in [24] covers broad ranges of parameters such as the yield strength, the slenderness ratio, and the aspect ratio. But, the simple design rule proposed in [24] may be too conservative for lower slenderness ratios, and also it is based on Bleich's formula [3].

Eurocode 3 [20] requires that the minimum stiffness obtained from linear elastic buckling theory be increased from 3 (for long panels) to 10 times (for short panels) to take account of postbuckling behavior following Kerensky et al. [25]. In AISC specification prior to 2010 and AASHTO LRFD specifications prior to 2007, the required flexural rigidities of the stiffeners were determined by Bleich's formula [3] considering only elastic buckling. AASHTO [18] for the first time added new provisions for postbuckling that were suggested by Kim et al. [26]. But stiffener design for postbuckling [18] is fundamentally based on Bleich's formula [3]. Similar provisions were newly specified in AISC [4].

In this study, a comprehensive method is suggested for the design of transverse stiffeners including end posts attached on shear web panels of steel I-girders. While AISC and AASHTO LRFD Specifications traditionally categorize web panels into three types for shear design, this study divides them into two types for the purpose of a simple and conservative design of transverse stiffeners. Although Bleich's formula [3] based on Stein and Fralich [2] has long been used for elastic buckling in AISC and AASHTO LRFD Specifications, not only its accuracy, but also its applicability to aspect ratios greater than 1.0 have yet to be examined. This study first investigates the moments of inertia of transverse stiffeners required to maintain straight nodal lines for elastic buckling through linear buckling finite element analysis, and develops new design equations. The results obtained by the new design equations are compared with those obtained by Bleich's formula [3]. Then, nonlinear finite element analyses are carried out in order to determine the moments of inertia of the transverse stiffeners required to maintain straight nodal lines during postbuckling. From the nonlinear FEA results, a new design equation is formulated for postbuckling based on the present design equations developed for elastic buckling. The new design equation for postbuckling is compared with those of AISC and AASHTO LRFD Specifications [4,5].

2. New categorization of web panels for stiffener design

Fig. 1 shows the shear strength curve of AASHTO [5] for web panels in which postbuckling strength can be used. The curve is divided into the three zones: (1) yield zone; (2) inelastic buckling zone; and (3) elastic buckling zone. In the yield zone, the elastic buckling strength is greater than the shear yield strength so that the stiffeners do not have to provide the nodal lines for elastic buckling. It is enough for the stiffeners to ensure that the web panel is able to develop the shear yield strength. The required flexural rigidity is, therefore, less than that for elastic buckling. For web panels falling into the elastic buckling zone, special attention should be given to the design of the stiffeners. Postbuckling of the web panel is often accompanied by severe out-of-plane bending deformations. All the classical failure theories of shear web panels including Basler [7] and Porter et al. [27] assume that the web panels are simply supported by the stiffeners. For the stiffeners to maintain straight nodal lines and to ensure that the web panel is able to develop its potential postbuckling strength, a much greater flexural rigidity is necessary than that for elastic buckling.

On the other hand, the inelastic buckling zone may have to be divided into two subzones for a better evaluation of the required rigidity as shown in Fig. 1: zone I and zone II. In zone I, the shear strength is less than the elastic buckling strength, which means the required flexural rigidity is less than that required for elastic buckling. In zone II, the shear strength is greater than the elastic buckling strength due to postbuckling but to a lesser degree than in the elastic buckling zone. The design of the stiffeners, however, can be simplified by dividing the inelastic zone in a different way. When the theoretical elastic buckling strength is greater than or equal to the yield shear strength, web panels are considered to be in the yield zone. Otherwise, web panels are considered to be in the elastic buckling zone. This will lead to a slightly conservative design.

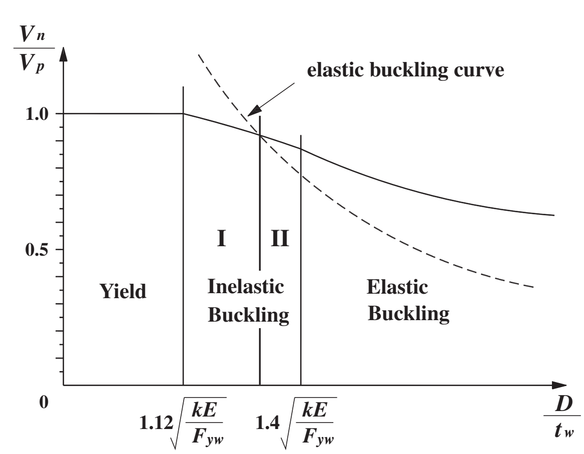

Fig. 1. AASHTO shear strength curve and subdivision of inelastic buckling zone.